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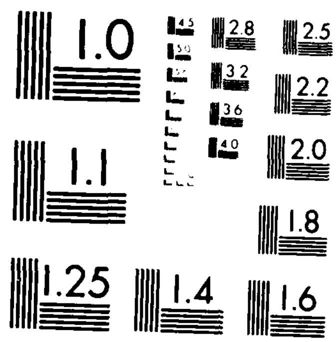

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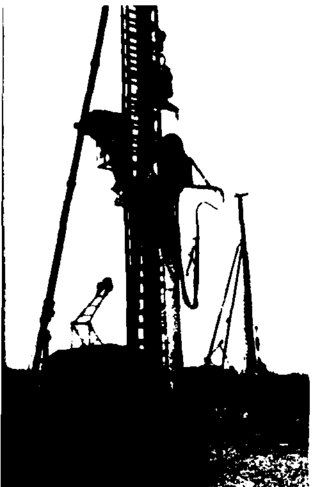

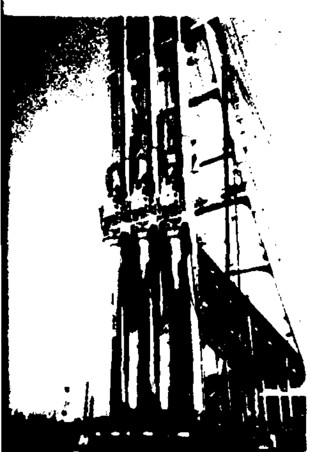

REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-GT-4

STATE OF THE ART FOR DESIGN AND CONSTRUCTION OF SAND COMPACTION PILES

by

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COVER PHOTOS:

TOP — Typical Sand Compaction Pile Construction Equipment.

BOTTOM — Typical Equipment Used To Construct Mammoth Compaction Poles Over Water.

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19 ABSTRACT (Continue on reverse if necessary and identify by block number)

Sand compaction piles can be used to improve marginal sites for stability, liquefaction, and settlement applications. They have been employed extensively in Japan for many years to improve land reclaimed from the sea. The advantages and disadvantages of using sand compaction piles are compared with other vibro-compaction techniques such as stone columns. Methods are described for construction of sand compaction piles on land and over water.

Design theories are given for the utilization of sand compaction piles at sites underlain by both cohesionless and cohesive soils. For sites underlain by cohesionless sands, procedures are presented for estimating the increase in standard penetration resistance in both the sand compaction pile and the surrounding sand. Techniques are described for estimating stability and one-dimensional consolidation settlement of sites underlain by cohesive soils that have been improved with sand compaction piles. Finally, typical applications of sand compaction piles are described, and practical design criteria and practices are given.

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PREFACE

The original research for this report was performed for the US Department of Transportation, Federal Highway Administration, under a contract to the School of Civil Engineering, Georgia Institute of Technology. The final report presented herein was prepared under Contract No. DACW39-85-M-2358 with the US Army Engineer Waterways Experiment Station (WES) during the period July 1985 to December 1986. The investigation was conducted under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program Work Unit 32275, "Remedial Improvement of Liquefiable Foundations." Mr. Authur H. Walz, Headquarters, US Army Corps of Engineers (HQUSACE), was REMR Technical Monitor.

The REMR Overview Committee, consisted of Mr. John R. Mikel (DAEN-CWO-M), Mr. Bruce L. McCartney (DAEN-CWH-D), and Dr. Tony C. Liu (DAEN-ECE-D). Coordinator for the Directorate of Research and Development was Mr. Jesse A. Pfeiffer, Jr. (DAEN-RDC), and the REMR Program Manager was Mr. William F. McCleese, Concrete Technology Division, Structures Laboratory, WES

Appreciation is expressed to the Federal Highway Administration for granting special permission to allow the publishing of this report. Mr. A. F. DiMillio was project manager, and Mr. Jerry DiMaggio was Technical Monitor for the Federal Highway Administration. Mr. Richard H. Ledbetter, Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), was Technical Monitor for the WES, under the supervision of Dr. Arley G. Franklin, Chief, EEGD, and under the general supervision of Dr. William F. Marcuson III, Chief, GL. Appreciation is extended for Mr. Ledbetter's help and careful review of the manuscript. Special thanks also go to Dr. P. F. Hadala, Assistant Chief, GL, for his thorough review of the manuscript.

The contents of this report reflect the views of the author who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the US Army Engineer Waterways Experiment Station. This report does not constitute a standard, specification, or regulation.

The design and construction practices for sand compaction piles described in this report are felt to be representative of those presently followed in Japan. These practices, as described, were developed from discussions with

Japanese and Taiwanese engineers, contractors, and equipment manufacturers along with field inspections and a review of the literature. Thanks are given to all the engineers and organizations, too numerous to acknowledge separately, who made valuable contributions to this study. Special acknowledgement is given to Mr. Tony Sullivan of Kencho, Inc. and Mr. Mizutani of Kensetsu Kikai Chosa Co., Ltd. for making detailed arrangements for the inspection trip to Japan and Taiwan.

COL Dwayne G. Lee, CE, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

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CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres
horsepower (550 foot-pounds (force) per second)	745.6999	watts
inches	25.4	millimetres
pounds (force)	4.448222	newtons
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per square foot	4.882428	kilograms per square metre
pounds (mass) per square inch	0.0703	kilograms per square centimetre
tons (2,000 pounds, mass)	907.1847	kilograms

STATE OF THE ART FOR DESIGN AND CONSTRUCTION OF SAND COMPACTION PILES

PART I: INTRODUCTION

1. Sand compaction piles are one potential method for improving marginal sites for stability, liquefaction and settlement applications. They also act as drains for static loading and accelerate primary consolidation. Sand compaction piles have been used extensively in Japan and Taiwan primarily for stability purposes in reclaiming land from the sea. Sand piles are similar to sand compaction piles but are not densified to as high a degree. Sand piles have been used in the United States primarily for sand drains to accelerate consolidation settlement but in recent years have been generally replaced by wick drains. Sand compaction piles, as defined in this report, have apparently not been used in the United States.

Advantages and Disadvantages of Sand Compaction Piles

2. Table 1 summarizes the advantages and disadvantages of sand compaction piles compared with several types of stone columns. The primary advantages of sand compaction piles over stone columns are (a) sand, which is often considerably cheaper than stone, is used in construction, (b) construction of the sand column is extremely fast, (c) the hole is fully supported by a casing during construction that eliminates the possibility of hole collapse, and (d) the possibility of intrusion and/or erosion of surrounding soil into the sand column is significantly reduced compared with stone columns; whether movement of particles can occur depends upon the gradation of both the sand compaction pile and the surrounding soil. Sand compaction piles are a possible solution for strengthening pervious embankment foundations found to be susceptible to liquefaction or impervious foundations susceptible to stability problems, for example, during an earthquake.

3. Sand compaction piles also have several important disadvantages that must be carefully considered during design. The primary disadvantages of sand compaction piles are: (a) they have a lower angle of

Table 1

Summary of Applications, Advantages, and Disadvantages of Selected Vibro-Ground Reinforcement Techniques

METHOD		APPLICATIONS	ADVANTAGES	DISADVANTAGES
Uncased Hole Construction - Stone Columns	Vibro-replacement (wet)	Very soft to firm cohesive soils[3] $150 \le c \le 1500$ psf; Silty sands ($<15\%$ fines); method used for most stability settlement, liquefaction applications	Conventional Method; More positive hole support than dry method; less hole smear than other methods; high k may allow u dissipation during earthquake; typical 20-60 tons design capacity; higher capacity in silty sands.	Some uncertainty exists concerning support of hole in very soft to soft soils; not suitable in peat layers $>$ 1 column dia.; must use large stone which may be expensive; must dispose of silty effluent; not suitable if sensitivity > 6; intrusion might occur.
	Vibro-displacement (dry)[1]	Partially saturated cohesive soils; $400 \le c \le 1000$ psf[3]; use where environmental restrictions exclude vibro-replacement; use in partially saturated clay fills; stability, settlement and liquefaction applications	No silty water to dispose; may be slightly cheaper than wet (smaller dia.); 15-40 ton design	Smaller load capacity and dia. than wet method; limited to partially saturated, better soils which will not collapse; more smear than wet method
Cased Hole Construction	Rigid Concrete Column[1]	Cohesive soils with 100 psf $\le c \le 800$ psf[3,4]; loose to firm silty sands and silts; use where peat is present; use in weak soil, $c \le 300$ psf; stability and settlement applications	No silty water disposal problem if dry; Good in organic soils and peat; Fully supported hole; small settlement; about the same cost as conventional stone column; fast; high load capacity up to 40-80 tons	Column does not act as drain; for stability applications evaluate local bearing failure of soft soil behind column using lateral load test; Column more brittle than conventional stone column; Use a granular distribution blanket and fabric above columns; for stability support entire embankment to prevent large bending moments in pile
	Stone Compaction Pile[1]	Cohesive soils with 200 psf $\le c \le 800$ psf[4]; very loose to firm silty sands, $N \le 15$; stability, settlement and liquefaction applications	No silty water disposal problem; fully supported hole; fast; can use finer gradation in very soft soils to prevent intrusion; 15-40 tons	Smaller dia. than vibro-replacement column; smear from advancing casing gives lower horizontal permeability than stone column; must use special bottom-feed rig; Dia. stone $\lesssim$ 1.5 in.
	Sand Compaction Pile[2]	Sands, silty sands; cohesive soils with $c > 100$ psf; use primarily for stability and tank/stockpile type applications; liquefaction	No silty water disposal problem; hole fully supported; uses cheap, local sand; very soft soil will not penetrate column; method is very fast; accelerates consolidation settlement; 15-35 tons	Smear from advancing casing reduces horizontal permeability; u not dissipated during earthquake; consider local bearing failure in very soft soils[5]; more settlement and lower strength than stone column

Notes:
1. A pull-down type rig is used to construct rigid concrete columns, stone compaction piles and sand compaction piles; vibro-displacement (dry) columns can also be constructed with a pull-down rig.
2. Sand compaction piles can be constructed using a pull-down type rig, by ramming or the Japanese method using a hollow pile with a vibrator at the top.
3. Predrilling may be used to advance hole in stiff soils with $c > 1000$ psf.
4. The vibrator can be run wet (with water jets operating) and penetrate to 1000 psf soil.
5. Local bearing failure in very soft soils is less of a problem for sand compaction piles (which carry less total load than stone columns).

internal friction and a lower stiffness than stone columns; hence in general a larger percentage replacement of weak soil is required using sand compaction piles; (b) driving the casing through a clay layer causes "smear" along the boundary of the column that reduces lateral permeability and hence their effectiveness as a drain. Nevertheless, an important decrease in time for primary consolidation occurs when sand compaction piles are used, and (c) sand compaction piles do not have sufficiently high permeability to function as effective vertical drains during earthquakes. Use of a sand compaction pile would increase the relative density of pervious native soils between the piles, the amount of improvement depending upon the pile spacing, method of construction and other factors. Also the strength of the sand compaction pile would increase the resistance to failure should liquefaction occur.

Sand Compaction Piles in Japan

4. The need in Japan to reclaim extensive areas from the sea lead to the first construction of sand piles between 1830 and 1850 (Ichimoto, 1980). Early in the 20th century a technique for compacting the sand pile was developed which was apparently somewhat similar to the process now used for constructing Franki type concrete piles. Today, sand compaction piles are usually constructed by driving a pipe having a special end restriction through a loose to firm sand, or very soft to firm silt or clay stratum using a vibrator located at the top of the pipe. Sand compaction piles generally have a diameter varying from 24 to 32 in.,* but may be as large as 80 in. Typically 5- to 7-ft sand compaction pile spacings are used in Japan. During or immediately after driving, the pipe is filled with sand. The sand is then densified by repeatedly raising and lowering the vibrating pipe as it is withdrawn from the ground. Several modifications of this procedure which can be described in Japanese terminology are mammoth compaction piles and strong sand piles. Mammoth compaction piles are quite similar to sand compaction piles except they are usually constructed over water using larger equipment. Strong sand piles are

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* A table of factors for converting non-SI units of measurement to SI (metric) units presented on page 4.

installed using the same procedure as for sand compaction piles but are further densified using a horizontal vibrator at the bottom of the casing. Sand compaction piles and mammoth compaction piles are the most commonly used sand pile techniques in Japan for improving poor sites.

5. In Japan over 160,000,000 ft of sand compaction piles and sand drains were constructed during the last 20 years by Fudo Construction Co. alone; undoubtedly this is just a small fraction of the total length constructed during this time. Probably the greatest use of sand compaction piles is to improve hydraulic sand fills and/or the underlying weak natural soils. Sand compaction piles in Japan are used primarily to prevent stability failures, decrease the time of consolidation and prevent liquefaction failure. Sand compaction piles are also used in Japan to reduce settlement although this appears usually to be a secondary objective since preloading is generally utilized. The distribution of uses of sand compaction piles by Fudo Construction Company is given in Figure 1; this summary, based on the results of the fact-finding trip, is felt to be generally representative of the overall use of sand compaction piles in Japan. Over 80 percent of the sand compaction piles are used to support stockpiles of heavy materials and various types of tanks and embankments for roads and railways. Only 4 percent of the sand compaction piles are used in Japan to support buildings and warehouses. The extent of use in Japan, if any, of sand compaction piles in dams, locks and levees is not known.

PART II: CONSTRUCTION OF SAND PILES

Sand Compaction Pile Construction

6. The equipment typically used to construct a sand compaction pile is shown in Figures 2 and 3 and the construction sequence in Figure [illegible]. For constructing sand compaction piles, a 4.5- to 6-ton hydraulic or electric vibrator is attached to the top of a 16- to 24-in. diam steel pipe. The pipe casing is slightly longer than the desired length of the sand compaction pile so as to protrude out of the ground after reaching the design depth. The pipe fully supports the surrounding soil at all times during construction. A description of how native soil is prevented from entering the pipe is subsequently described in paragraph 12. Water jets are sometimes used on the outside of the pipe when layers are encountered that have a standard penetration test (SPT) resistance greater than about 15 to 20. Water jet pressures up to 1200 psi have been used to aid driving piles using vibratory hammers (Hayashi, 1981).

7. The casing with attached vibratory hammer is suspended from a crawler crane and is guided by leads. A 35- to 40-ton crawler crane is used for constructing 60- to 65-ft-long sand compaction piles. A coil spring shock absorber is fastened to the top of the vibrating hammer to dampen the shock as the casing is pulled from the ground by the crane. During driving, the cable from the crane to the casing is kept slack so that the pipe-vibrator assembly is free floating. Proper pipe alignment is maintained by a guide attached to the vibrator that moves up and down the crane leads.

8. A low frequency, high amplitude vertical vibrator is used having a frequency of 500 to 600 cpm and amplitude during idling of 0.6 to 0.7 in. The amplitude is defined as one-half the total tip movement. The commonly used vibrators are driven by 120 to 160 hp motors and have unbalanced forces varying from 90,000 to 135,000 lb as summarized in Table 2. Large vibrators having greater than 120 hp are usually used for only the construction of mammoth compaction piles over water, described in paragraph 16.

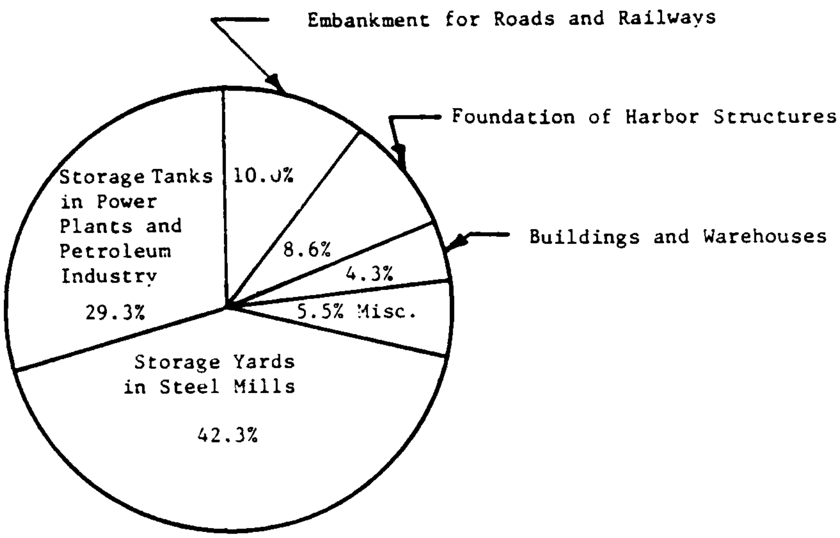

Figure 1. Distribution of sand compaction pile applications (after "Compozer System," Fudo Construction Co., Ltd., anonymous (undated), Tokyo, Japan

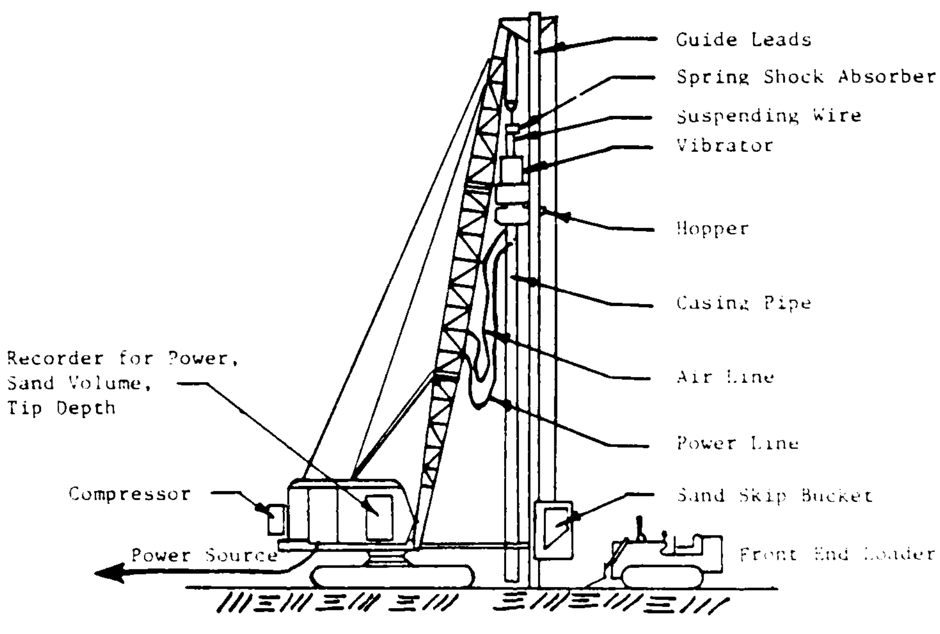

Figure 2. Typical equipment used to construct a sand compaction pile in Japan (after Tanimoto, 1973)

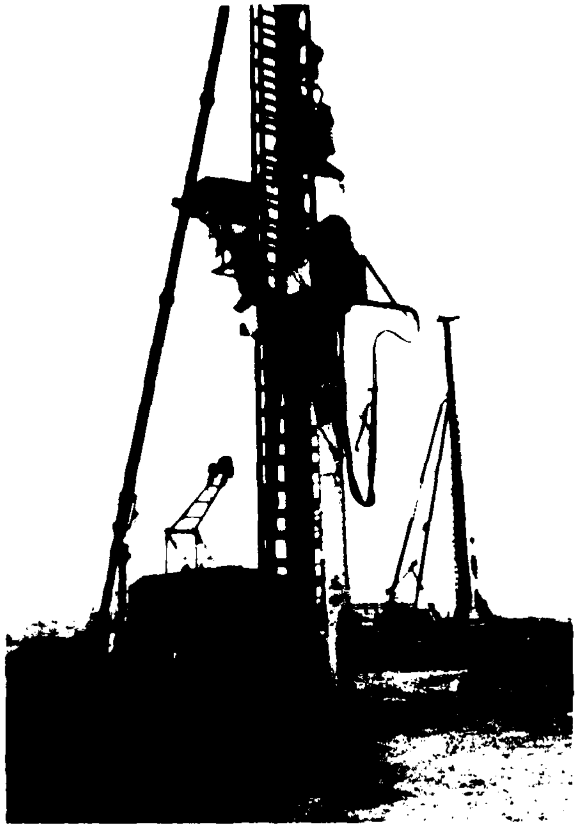

Figure 3. Photograph of typical sand compaction pile construction equipment used in Japan and Taiwan

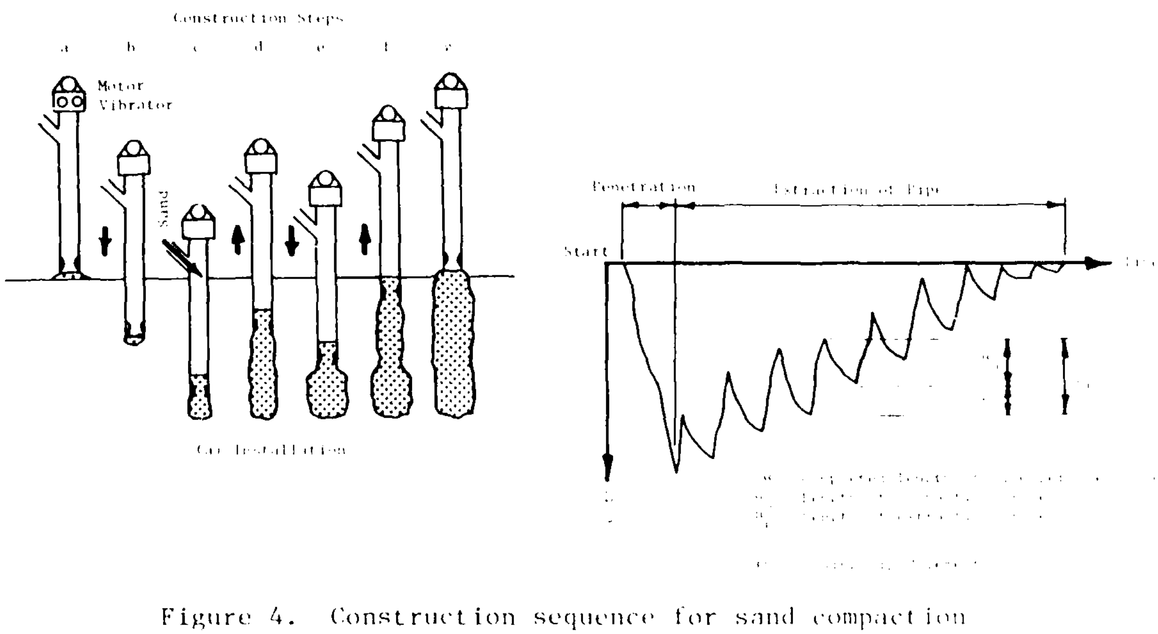

Figure 4. Construction sequence for sand compaction piles (after Tanimoto, 1973)

Table 2

Typical Japanese Vertical and Horizontal Vibrators Available

For Sand Pile Construction*

Manufacturer	Ground Improvement Application	Motor Output KW(HP)	Vibration Frequency (cpm)	Centrifugal Force (metric ton)	Amplitude (idling) mm (in.)	Suspended Weight (kips)	Power Source KVA
KENSETSU KIKAI CHOSA CO., LTD.							
Vibratory Hammer+ KM2-12000A	Driving Casing (N<40-50)	90 (120)	510	34.9	22.1 (7/8)	12.0	300
Vibratory Hammer VM2-25000A	Driving Casing (N<40-50)	150 (200)	620	65-107	19.8 - 32.9 (13/16 - 1-5/16)	16.7	600
Vibrofloat+ (Vilot) KSV-3000A	Densify Sand SSP Piles**	11 (8.5) Electric	1800	-	-	1.10	-
Vibrofloat++(Vilot) KSV-12000A	Densify Sand SSP Piles**	15 (20) Electric	1690@ 60 Hz	2.7 - 3.8	-	5.28	200-400
Vibrofloat++(Vilot)	Densify Sand SSP Piles**	30 (40)	1150	9.0	-	-	-

* These vibrators are available in the United States from Kenchou, Inc., Hayward, California.
** Strong sand piles
\+ Vertical vibrator
\++ Horizontal vibratory

Filling steel casing

9. To minimize construction time, the steel casing is usually filled with sand as it is being driven down so that extraction of the casing can begin immediately upon reaching the required depth. After filling the skip with a front end loader (Figure 2), the sand is mechanically lifted and dumped into the hopper located at the top of the pipe as the pipe is being vibrated down (Figures 2 and 3). The skip bucket is guided by the leads suspended from the crane. This very efficient method of construction was observed at all construction sites visited in both Japan and Taiwan. If an automatic skip bucket is not available, the hopper at the top of the casing can be filled using the front-end loader after it has almost reached the required depth. This alternate approach requires less mechanical equipment, but the sand compaction pile production rate is considerably reduced. Typical production rates are summarized in paragraph 13.

Sand removal

10. Upon filling the casing with sand, a 40- to 70-psi air pressure is applied to the top of the sand column. To develop the required air pressure on top of the sand, a pressure of about 100 psi is developed at the compressor. A special valve (Figure 5) is used to introduce the air pressure to the sand yet keep the hopper sealed from the atmosphere. The air pressure on the top of the sand prevents soft soil from flowing into the pipe and helps to force the sand out during withdrawal. Water is usually not utilized during the construction of a sand compaction pile. If the sand is dry, however, contractors frequently add some water to the sand to prevent it from sticking in the tube due to arching.

Sand densification

11. A sand compaction pile is constructed using a stroking motion of the casing as it is withdrawn (Figure 4). The casing is first pulled up 6 to 10 ft. using the crane, and then vibrated back down 3 to 7 ft. This up-and-down stroking motion is repeated until the casing has been completely withdrawn from the ground. The stroking motion apparently plays an important role in achieving a strong, dense sand compaction pile, and in densifying the surrounding sands. As the sand compaction pile is constructed, the depth of the casing tip, power consumption and approximate volume of sand consumed are usually continuously recorded. If a stroking motion is not used, the resulting column is called simply a sand pile.

Closing casing end during driving

12. During driving the lower end of the casing is closed using one of the special end cover assemblies shown in Figure 6. Figure 6(a) shows the tip used by the contractors at the sites visited in Japan and Taiwan. This tip consists of several hinged metal fingers that are manually pushed together to form a closed end before driving begins. As the casing is withdrawn the fingers open out under the action of gravity (Figure 7) allowing the sand to flow out. Fudo Construction Company uses a constriction in the end of the pipe relying on friction and the weight of the sand to maintain equilibrium during driving (Figure 6(b)). An end plate hinged in the middle is also sometimes used in Japan (Figure 6(c)).

Summary

13. Construction of sand compaction piles using the equipment and procedures described in this section is fast and efficient. Using this process a 50-ft sand compaction pile can be constructed in about 20 min with average daily production rates being about 20 piles. About 45 sand compaction piles per day on the average can be constructed using 20-ft-long piles.

Strong Sand Piles

14. Several modifications to the basic method are sometimes used in Japan to construct sand compaction piles. The strong sand pile method, developed in 1973, is used in soft clays in the same way as sand compaction piles.* To construct a strong sand pile, a casing is first driven by a vertical vibrator attached to the top of the casing using similar equipment and procedures previously described for sand compaction piles. In constructing strong sand piles, however, a horizontal vibrator is placed just below the bottom of the casing as shown in Figure 8. Because of the use of the horizontal vibratory (called a Vilot) at the tip of the casing, a higher degree of densification of the sand is reported by Hayashi 1981 to be obtained than for sand compaction piles.

15. During or after driving of the casing, sand and water are introduced through the sand inlet at the top of the casing (Figure 8). Adding water to the sand in this method helps to achieve a higher density of the sand during vibration, and also allows the sand to flow out of the

* "SVS Method Technical Information," Kensetsu Kikai Chosa Co., Ltd., anonymous (undated), Osaka, Japan.

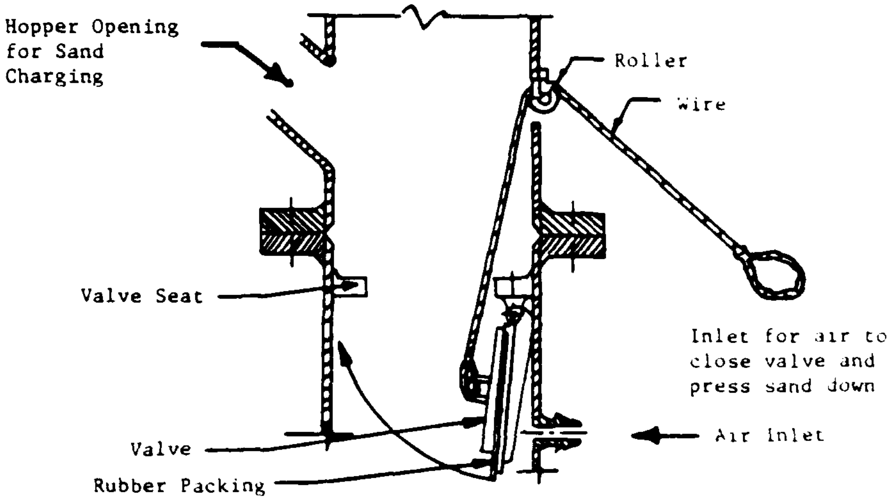

Figure 5. Special valve used to seal the casing when air pressure is introduced

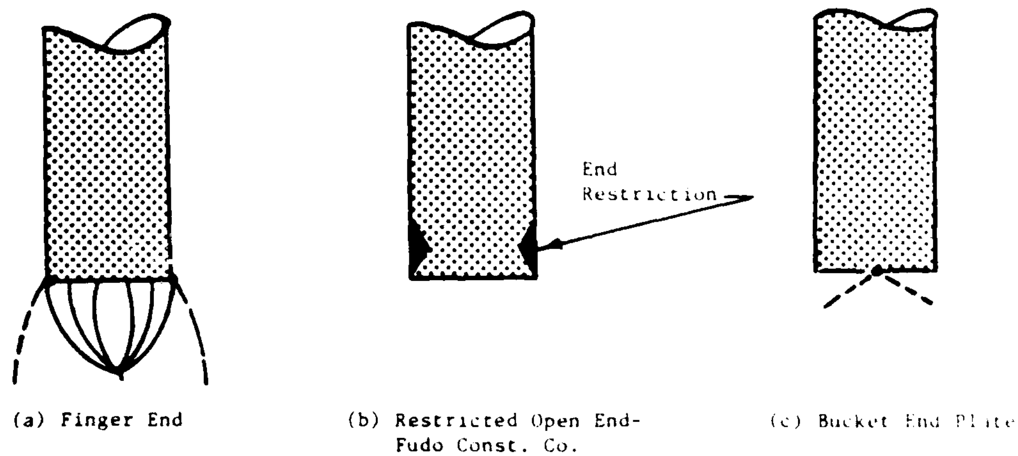

Figure 6. Methods used to prevent soil from entering casing during driving

Figure 7. Photo of finger end which prevents soil from entering casing during driving

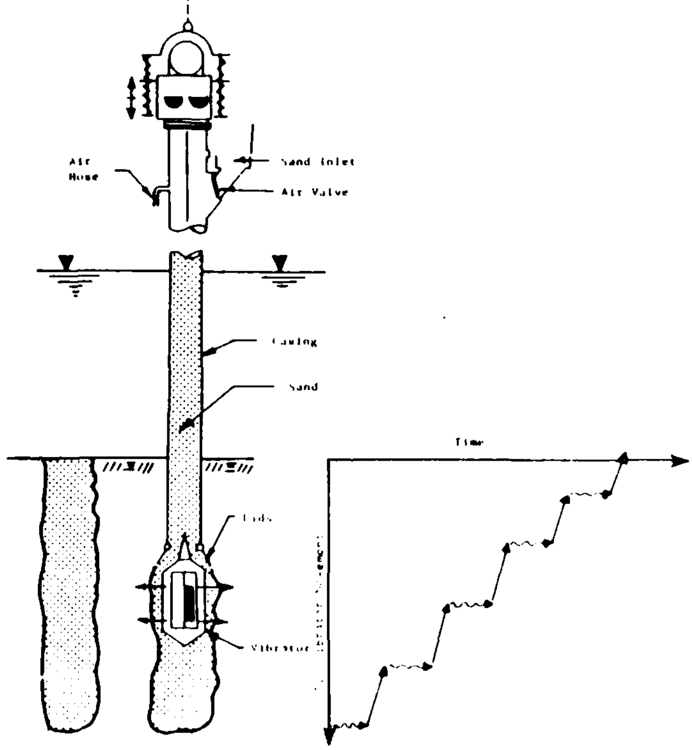

Figure 8. Construction of strong sand compaction piles over water

casing more easily. Sand is usually supplied to the casing using the same automatic skip bucket employed for sand compaction piles. On land the skip bucket is filled with a front-end loader. When working over water from a barge, a belt conveyor is used to fill the bucket.

16. After filling the casing with sand, the inlet is closed and 40 to 70 psi of compressed air is introduced to the top of the sand. The vibrator at the bottom of the casing is then actuated causing a horizontal and circular vibration. As the casing is gradually extracted, the lids at the bottom of the casing open outward and sand flows out forming the pile (Figure 8). The horizontal vibrator operates continuously during extraction. At selected intervals extraction of the casing may be stopped to achieve additional densification of the sand. A stroking motion during extraction is apparently not generally used.

Mammoth Compaction Piles

17. Mammoth compaction piles are constructed in Japan in depths of water up to 100 ft to provide a foundation on which to reclaim land from the sea. The stabilization of very soft ocean sediments with mammoth compaction piles is accomplished using materials and construction techniques similar to the sand compaction piles constructed on land. Mammoth compaction piles are routinely constructed in Japan having diameters of 24 to 79 in. and lengths up to 160 ft below sea level.

18. From two to four mammoth compaction piles are constructed simultaneously from a large barge such as the one shown in Figure 9. Each casing is driven to the required tip elevation using a large vertical vibrator. The vibrator is mounted at the top of the casing and typically driven by a 160- to 400-hp motor. In very soft sediments the casings are sometimes pulled down by a cable rather than vibrated. The remainder of the construction operation is similar to sand compaction piles which have been previously described.

General Considerations

19. Sand pile diameters of 24 to 47 in. are usually constructed using casing diameters of 20 to 32 in., and a 120- to 160-hp vertical vibrator.

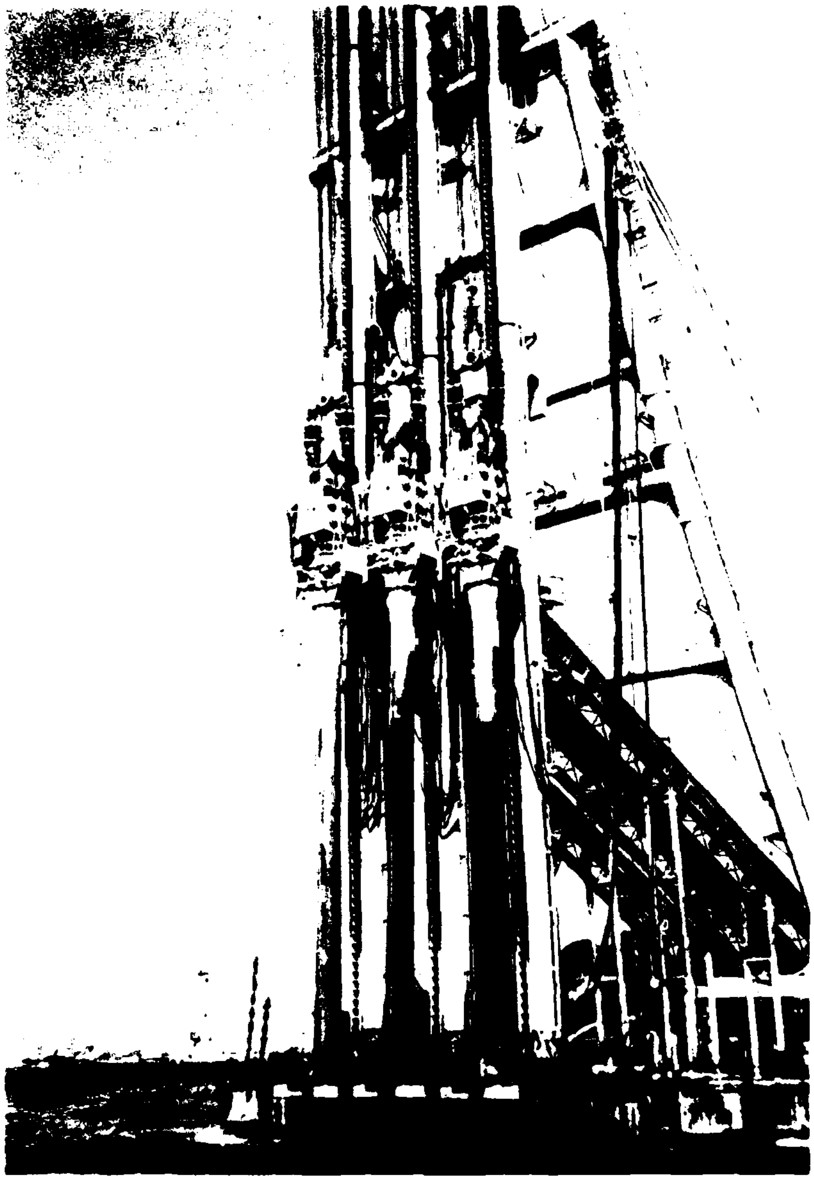

Figure 9. Photo of typical equipment used to construct mammoth compaction piles over water

Sand pile diameters of 32 to 79 in. are constructed using 24- to 47-in. casing and 160- to 400-hp vertical vibrators. A 10- to 40-hp horizontal vibrator is sometimes used at the bottom of the casing. Table 2 gives specifications of horizontal vibrators (Vilots) used to construct strong sand piles. At the present time, however, the strong sand pile method is not nearly as popular in Japan as sand compaction piles.

20. The diameter and density of both sand compaction piles and strong sand piles can be varied by changing the (a) casing diameter, (b) extraction rate, (c) compressed air pressure, (d) time of vibration and, (e) size of vibrator. Further, a pile diameter varying with depth can be obtained by changing (b) through (d) during casing extraction. During construction the diameter of the sand pile, volume of sand supplied and discharged, casing tip elevation, and the power (amps) used by the vibrator are frequently all automatically recorded.

PART III: DESIGN THEORY FOR SAND COMPACTION PILES

Introduction

21. In this section the theory for designing sand compaction piles and mammoth compaction piles is presented as presently followed in Japan. Sites underlain by both sand and cohesive soils are considered. The methods presented can also generally be used for strong sand piles which are constructed as previously described using both a vertical and horizontal vibrator.

Area-Replacement Ratio

22. The volume of soft clay or loose sand replaced by sand is one of the most important factors in improving weak ground using sand compaction piles, strong sand piles, or mammoth compaction piles (and also stone columns). To quantify the amount of soil replacement, define the area replacement ratio a_s as the fraction of soil tributary to the pile replaced by the sand compaction pile:

$$a_s = A_s/A \tag{1}$$

where A_s is the cross-sectional area of the completed sand compaction pile and A is the total area tributary to the sand compaction pile, as illustrated in Figure 10. The area replacement ratio can be expressed in terms of the diameter and spacing of the sand compaction pile as follows:

$$a_s = C_1 \left(\frac{D}{s}\right)^2 \tag{2}$$

where

s = center-to-center spacing (pitch) of the sand compaction piles

C_1 = a constant dependent upon the sand pile pattern used; for a square pattern $C_1 = \pi/4$ and for an equilateral triangular pattern $C_1 = \pi/(2\sqrt{3})$

D = diameter of the completed sand compaction pile (not the diameter of the casing)

23. For an equilateral triangular pattern of sand compaction piles, which are more frequently used in the United States for stone columns, the area-replacement ratio is then

$$a_s = 0.907 \left(\frac{D}{s}\right)^2 \qquad (3)$$

In working with ground improvement using sand compaction piles (or stone columns), it is important to think and work in terms of the area-replacement ratio. Area-replacement ratios for sand compaction piles constructed on land are typically 0.4 to 0.5 (refer to paragraph 55).

Sites Underlain by Sand

24. For sites underlain by sands the required sand compaction pile spacing (pitch) and diameter can be estimated using the theoretical approach described by Aboshi et al. (1979; Tanimoto (1973); Ichimoto (1980); and others.* This approach is based on the fact that the strength and settlement properties of a cohesionless soil are primarily determined by relative density. The importance of stress history, grain size, gradation, angularity, and other characteristics should, of course, not be forgotten. For each cohesionless soil a single void ratio is associated with each value of relative density. If the required increase in relative density of a loose sand can be determined from stability, settlement, or liquefaction considerations, the required reduction in void ratio can be readily estimated using basic relationships described in this section.

25. Assume that sand compaction pile construction causes, during installation, only lateral displacement of the loose sand. After making this assumption, the size of the sand pile required to cause the decrease in void ratio by displacement of the loose sand to the desired value can then be calculated.

26. In Japan, as elsewhere, many practicing engineers use standard penetration test results as the basis for making stability and settlement

* SVS Method Technical Information, "Kensetsu Kikai Chosa Co., Ltd., anonymous (undated), Osaka, Japan; "Sand Pile Construction Using Vibrating Pile-Driving Equipment, "Kensetsu Kikai Cosa Co., anonymous (undated), Osaka, Japan.

estimates. Sand compaction pile design can be based on either standard penetration resistance or relative density since the two quantities can be related to each other. The design procedure requires the standard penetration resistance value N to be corrected for the effect of overburden pressure using for example the results of Holtz and Gibbs (1957) or Ogawa and Ishido (1965).

Design approach

27. To develop a practical design method, assume the total volume tributary to a sand compaction pile remains constant during the site improvement work. Also, neglect any increase in relative density caused by vibration as the casing is driven, and assume the loose sand is only displaced laterally away from the sand pile during construction. The relationship between the required volume of the sand compaction pile and the change in void ratio of the in situ sand can be derived using the notation and relationships shown in Figures 10 and 11. Referring to Figure 11, let the change in volume of the in situ sand equal the volume of the sand compaction pile $\bar{S}$ giving

$$\bar{S} = V_o - V_1 = V_s(1+e_o) - V_s(1+e_1) \qquad (4)$$

which simplifies by cancellation to

$$\bar{S} = V_s(e_o - e_1) \qquad (5)$$

The volume of solids equals the volume of voids divided by the initial void ratio

$$V_s = V_v/e_o \qquad (6)$$

Then substituting equation (6) into (5) gives

$$\bar{S} = \frac{V_v}{e_o}(e_o - e_1) \qquad (7)$$

28. Now, consider a thin, horizontal slice of sand compaction pile and tributary soil having a unit thickness $\ell = 1$, as illustrated in Figure 10. Referring to Figure 11a, the total volume of material originally present is

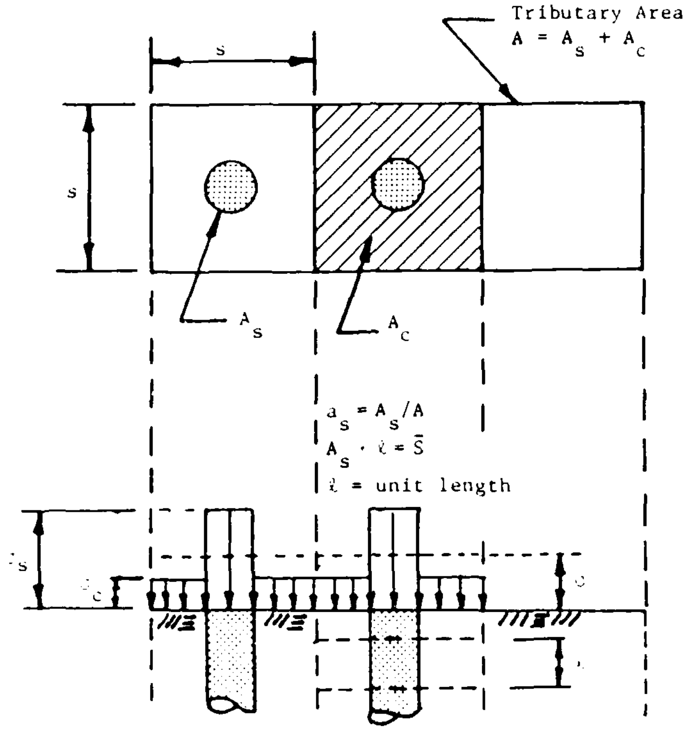

Figure 10. Geometry and stress state associated with a soil reinforced by sand compaction piles

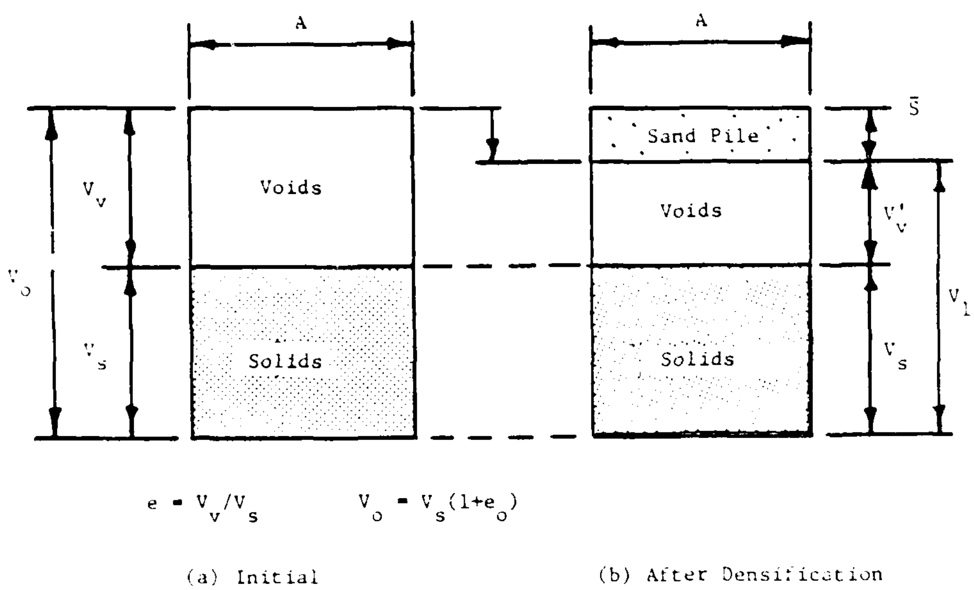

Figure 11. Volume block diagram of in situ sand before and after sand pile construction

$$A \times \ell = V_v + V_s = V_v + \frac{V_v}{e_o} \qquad (8)$$

Solving Equation (8) for the original volume of voids V_v and letting $\ell=1$ gives

$$V_v = \left(\frac{e_o}{1+e_o} \right) A \qquad (9)$$

Now divide each side of Equation (7) by the total area A, and replace V_v by Equation (9). This gives upon simplification the ratio of volume of sand compaction pile to the total volume for a unit increment of length

$$\frac{\bar{S}}{A} = \frac{e_o - e_1}{1 + e_o} \qquad (10)$$

where

$\bar{S}$ = volume of sand compaction pile per unit length, $\ell = 1$

A = total tributary area to one sand compaction pile

e_o = initial void ratio of the loose sand

e_1 = void ratio of the loose sand after sand compaction pile construction

Since the volume of sand compaction pile $\bar{S}$ is defined for a unit length $\ell = 1$ of construction, the area-replacement ratio, which is defined by Equation (1), equals $\bar{S}/A$ giving

$$a_s = \frac{e_o - e_1}{1 + e_o} \qquad (11)$$

where

e_o = initial void ratio of loose sand before improvement

e_1 = final void ratio of loose sand after improvement

29. The above expression can be changed to a more useful form for design by considering a unit length of sand pile construction. For a square sand compaction pile grid having a spacing s, the total volume (Figure 11(a)) is equal to $V_o = s \times s \times \ell$. For a unit length $\ell = 1$, solving for sand compaction pile spacing s gives for a square pattern

$$s = \sqrt{V_o} = \sqrt{\frac{1 + e_o}{e_o - e_1} \cdot \bar{S}} = \sqrt{\frac{1 + e_o}{e_o - e_1} \cdot \frac{\pi D^2}{4}} \qquad (12a)$$

and similarly for an equilateral triangular pattern

$$s = \frac{2}{3}\sqrt{V_o} = 1.08\sqrt{\frac{1 + e_o}{e_o - e_1} \cdot \bar{S}} = 1.08\sqrt{\frac{1 + e_o}{e_o - e_1} \cdot \frac{2\pi D^2}{4}} \qquad (12b)$$

where

s = spacing (pitch) of the sand compaction piles

V_o = total volume of soil (in situ sand plus sand compaction pile) tributary to the sand compaction pile per unit length of depth

$\bar{S}$ = volume of sand compaction pile per unit length of depth = $\pi D^2/4$

D = pile diameter

Due to waste and densification of the added sand during construction, the volume of loose sand brought to the site for sand pile construction must be greater than the volume of compacted sand pile. The ratio of volume of loose sand brought to the site to densified sand within the sand compaction pile has been found in Japan from experience to typically be between 1.2 and 1.4. In practice a ratio between 1.2 and 1.3 is frequently used.

30. The following trial and error design procedure can be used to determine the required sand compaction pile spacing (pitch) and sand volume:

1. Make a preliminary estimate of the required relative density after improvement of the in situ sand.

2. Select, based on the soil and loading conditions, a trial spacing (pitch). Typically this spacing is between 5.5 and 8 ft.

3. Estimate the initial and final void ratio of the in situ sand. The best approach would be to determine in the laboratory the actual relationship between void ratio and relative density for the sand encountered at the site. As an expedient alternate the lower portion of Figure 12 can be used as a guide in estimating the void ratio-relative density relationship. Three "typical" void ratio-relative density curves are given for varying uniformity coefficients C_u and effective grain sizes D_{60}. In Japan curves similar to Figure 12 apparently are frequently used for design.

4. Using the change in void ratio of the in situ sand obtained from Step 3, calculate the required volume of densified sand per unit

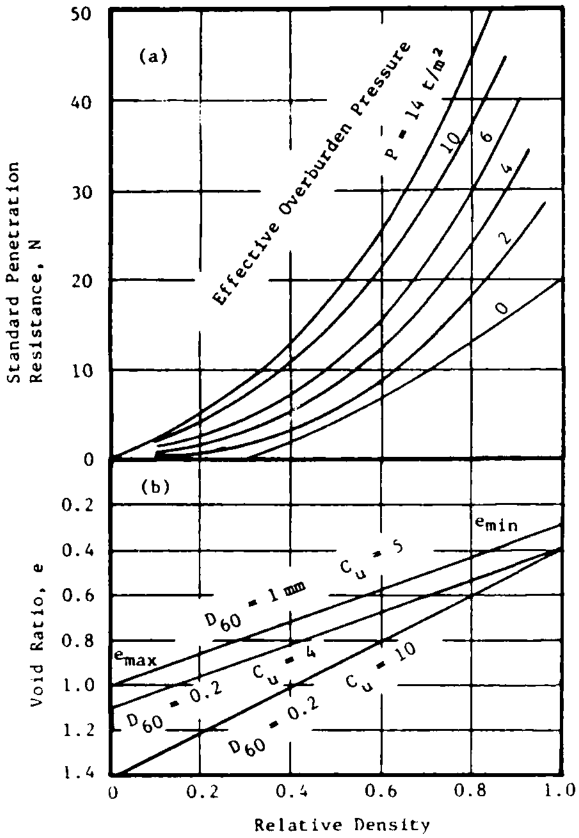

Figure 12. Relationship between relative density and penetration resistance or void ratio (after Tanimoto, 1973)

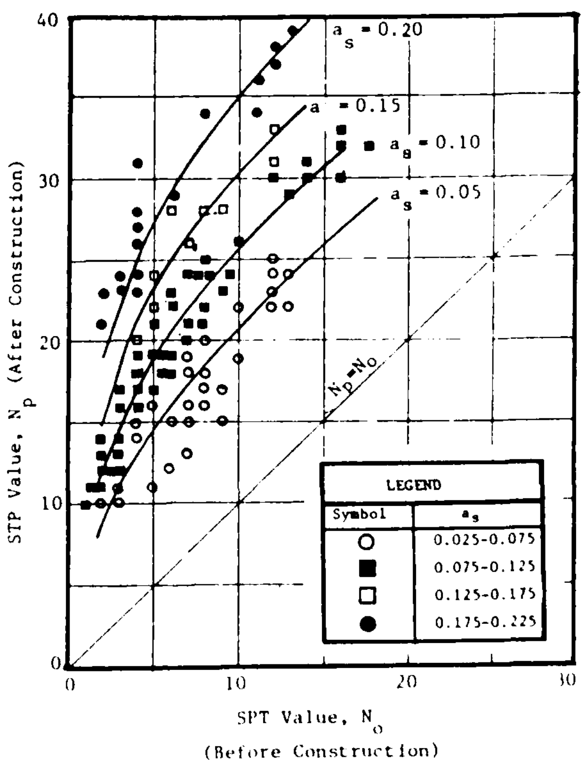

Figure 13. Effect of sand compaction pile construction on SPT value at center of sand pile ("Sand Pile Construction Using Vibrating Pile-Driving Equipment," Kensetsu Kikai Chosa Co., Ltd., anonymous (undated), Osaka, Japan)

length of depth using either Equation (12a) or (12b) depending on the sand compaction pile pattern. Then calculate the required sand compaction pile diameter from the known sand volume $\bar{S}$ or using Equation (2). The required replacement ratio can be determined from Equation (11). The sand compaction pile diameter used in practice is usually between 24 and 32 in.

5. Estimate the required inside diameter of the casing to be used in constructing the sand compaction pile considering the initial relative density and characteristics of the soil and vibratory hammer-pile system. The constructed sand compaction pile diameter is usually taken as 1.5 to 1.6 times the inside diameter of the pipe.

6. If the required sand compaction pile and/or pipe diameter is not practical, select a new spacing (pitch). Refer to paragraph 55 for typical sand compaction pile and pipe diameters used in Japan. Then repeat Steps 1 through 6, considering the new area replacement ratio a_s and available equipment.

31. The above steps give a diameter of the sand compaction pile that is compatible with the final relative density of the in situ sand assumed in Step 1. Following Steps 1-6 does not, however, ensure an overall safe design. Therefore, the preliminary design must be checked and revised as necessary for safety with respect to stability, settlement, and liquefaction using appropriate engineering analyses.

Design based on SPT

32. An alternative approach used in Japan for designing sand compaction piles is based on previously observed increases in standard penetration resistance of sand compaction piles during construction. Field measurements made before-and-after sand pile construction show that the standard penetration resistance after construction significantly increases as the area replacement ratio a_s becomes greater (Figures 13 and 14). Figures 13 and 14 can therefore be used to estimate the required area-replacement ratio after selecting a final field value of standard penetration resistance. This empirical approach can be easily used to verify the results obtained from the theoretical method given previously.

Casing extraction

33. Sand compaction piles are densified as previously discussed using a stroking motion as the casing is extracted from the ground. The required

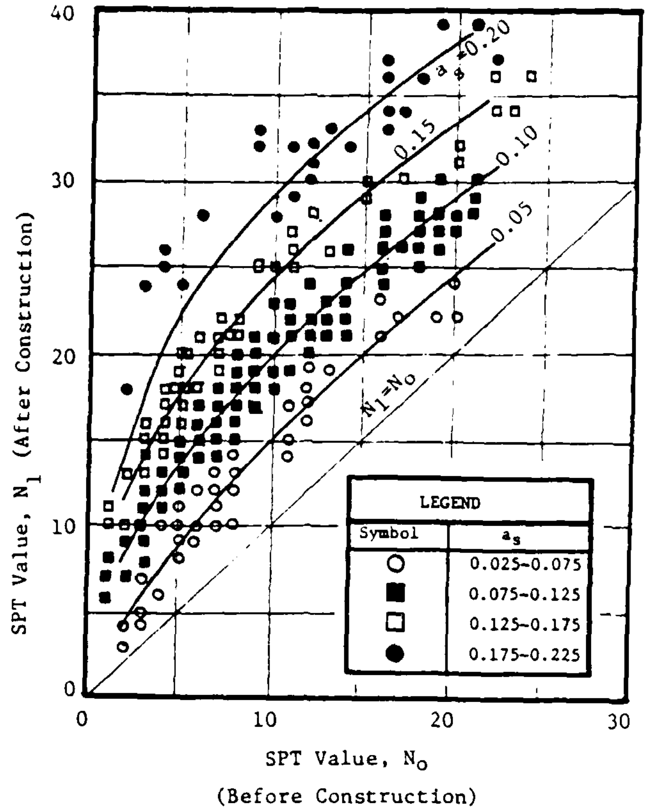

Figure 14. Effect of pile construction on SPT value one-half way between sand piles ("Sand Pile Construction Using Vibrating Pile-Driving Equipment," Kensetsu Kikai Chosa Co., Ltd., anonymous (undated), Osaka, Japan

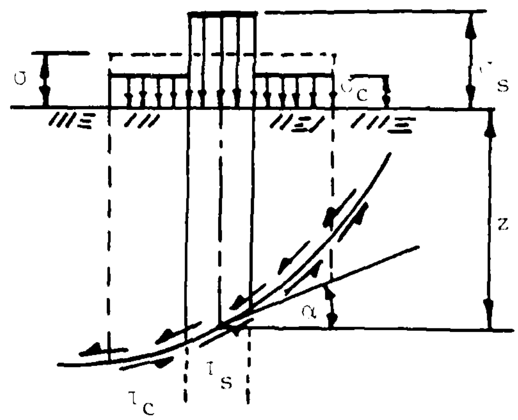

Figure 15. Notation used in stability analysis (after Aboshi, et al., 1979)

distance to redrive the pipe can as a rough approximation be estimated using the following empirical expression*

$$H_D = H_E \left(1 - n' \frac{A_p}{A_s}\right) \tag{13}$$

where:

H_D = distance casing must be redriven downward

H_E = distance casing is extracted before redriving

n' = empirical factor that can be taken as about 0.8

A_p = inside area of the casing

A_s = area of the completed sand compaction pile

More rational methods for estimating the distance to redrive the casing are apparently not presently used in Japan.

34. During the extraction-redriving cycle the quantity of sand discharged is apparently not exactly proportional to the extraction height H_E.* Because of its approximate nature, the above expression should be used together with standard construction practice for specifying values of H_E and H_D. Usually in practice the casing is extracted 6.5 to 10 ft and redriven about one-half that height. Fudo Construction Company typically extracts the casing 6.5 ft.

Average standard penetration resistance

35. The average weighted penetration resistance, $\bar{N}$ of the improved ground including the installed sand compaction pile can be estimated using the following expression*

$$\bar{N} = a_s N_p + (1 - a_s)\, N'_s \tag{14}$$

where

$\bar{N}$ = average weighted standard penetration resistance of the improved soil including the sand compaction pile

a_s = area-replacement ratio

N_p = standard penetration resistance of the sand compaction pile

N'_s = standard penetration resistance of the improved soil usually taken one-half the way between the sand compaction piles

* "Sand Pile Construction Using Vibration Pile-Driving Equipment," Kensetsu Kikai Chosa Co., Ltd., anonymous (undated), Osaka, Japan.

At sites underlain by sand, the average weighted standard penetration resistance is frequently used for design and given in specifications as the end product of sand compaction pile construction.

Sites Underlain by Cohesive Soils

Stress concentration

36. Sand compaction piles are frequently used in very soft cohesive soils to provide stability, accelerate consolidation, and reduce settlement. Both field and laboratory studies show that, upon loading, an important concentration of stress as shown in Figure 10 occurs in the relatively stiff sand compaction pile. The stress concentration occurs since the total settlement in the sand and clay is approximately the same (Aboshi, et al., 1979). This concentration of stress forms the basis of design for both stability and settlement of a sand compaction pile reinforced cohesive soil. The concentration of stress is expressed by a stress concentration factor n defined as

$$n = \sigma_s / \sigma_c \qquad (15)$$

where:

σ_s = vertical stress in the sand compaction pile

σ_c = vertical stress in the clay

37. The value of the stress concentration factor used in design has an important effect on both calculated settlement and stability. Typically, a stress concentration factor of about 3 to 5 is used in Japan based on past experience and field measurements (refer to paragraphs 64 and 65).

38. Consider the case of a very wide, relatively uniform loading σ applied to a large group of sand compaction piles having a square or equilateral triangular pattern. The average stress σ (or stress increase $\Delta\sigma$) which must exist at a given depth over the total tributary area for equilibrium of forces must equal

$$\sigma = \sigma_s \cdot a_s + \sigma_c (1 - a_s) \qquad (16)$$

where all the terms have been previously defined. Solving Equation 16 for the stress in the clay and sand using the stress concentration factor n gives

$$\sigma_c = \sigma/[1 + (n - 1)\ a_s] = \mu_c \sigma \tag{17a}$$

$$\sigma_s = n\ \sigma/[1 + (n-1)\ a_s] = \mu_s \sigma \tag{17b}$$

where μ_c and μ_s are the ratio of stress in the clay and sand, respectively, to the average stress over the tributary area. For a given set of field conditions, the stress in the sand and clay can be readily determined using Equations (17) upon assuming a value of the stress concentration factor.

Stability

39. In Japan one of the most important uses of sand compaction piles for site improvement is to prevent a stability type failure of wide fills, tanks, dikes, embankments, and heavily loaded storage yards constructed over very soft sediments. The stability failure might be either within recently placed hydraulic fill or the underlying soft native foundation soil. Frequently moderate to large preloads are used to limit settlements beneath tanks and heavy storage yards. The stability of an improved site reinforced with compaction piles is usually analyzed in Japan using a conventional circular arc stability analysis and the average weighted shear strength of the material within the tributary area of each pile.

40. Consider the stress state within a selected sand compaction pile at the depth where the circular arc intersects the centerline of the pile using the notation shown in Figure 15. The effective stress in the sand compaction pile due to its weight and any externally applied loading can be expressed as

$$\bar{\sigma}_z = \gamma_s' z + \Delta\sigma\ \mu_s \tag{18}$$

where

$\bar{\sigma}_z$ = vertical effective stress acting on the sliding surface of a sand column

γ_s' = unit weight of sand (buoyant if below the ground water table)

z = depth below the ground surface

$\Delta\sigma$ = stress increase at depth under consideration due to the embankment loading

μ_s = stress concentration factor for the sand column, Equation (17b)

41. Referring to Figure 15, the shear strength of the sand column can then following common United States practice* be expressed as

$$\tau_s = (\bar{\sigma}_z \cos^2\alpha) \tan\phi_s \tag{19}$$

where

τ_s = shear strength in the sand column

α = inclination of the shear surface with respect to the horizontal

ϕ_s = angle of internal friction of the sand column

The average weighted shear strength within the area tributary to the sand pile then becomes

$$\tau = (1 - a_s)\, \tau_c + a_s\, \tau_s \tag{20}$$

where:

τ = average weighted shear strength

τ_c = undrained shear strength of the cohesive soil

and the other terms have been previously defined.

42. The weighted average unit weight within the reinforced ground is used in calculating the driving moment

$$\gamma_{avg} = \gamma_s \cdot a_s + \gamma_c \cdot (1 - a_s) \tag{21}$$

where γ_s and γ_c are the saturated (or wet) unit weight of the sand and cohesive soils, respectively. In this approach the weighted shear strength and unit weight are calculated for each row of sand columns and then used in a conventional hand stability analysis. A more detailed consideration of methods for performing stability analyses of soils improved with granular columns is given elsewhere (Barksdale and Bachus, 1983a).

* In Japan apparently, $\cos\alpha$ rather than $\cos^2\alpha$ is at least sometimes used.

Increase in Shear Strength Due to Consolidation

43. The shear strength of a soft cohesive soil reinforced with sand compaction piles rapidly increases during and following construction of an embankment, tank, or foundation. The additional stress due to construction results in an increase in pore pressure causing consolidation of the soil accompanied by an increase in shear strength. The rate of embankment construction is frequently controlled to allow the shear strength to increase so that the required minimum safety factor with respect to a stability failure is maintained.

44. The undrained shear strength of a normally consolidated clay has been found to increase linearly with effective overburden pressure (Leonards, 1962). For this type cohesive soil the undrained shear strength can be expressed as

$$c = K_1 \cdot \bar{\sigma} \tag{22}$$

where:

c = undrained shear strength

K_1 = the constant of proportionality defining the linear increase in shear strength with $\bar{\sigma}$, i.e., $K_1 = c/\bar{\sigma}$

$\bar{\sigma}$ = effective overburden pressure

45. For a cohesive soil having a linear increase in shear strength with $\bar{\sigma}$, the increase in undrained shear strength Δc_t with time due to consolidation can be expressed for sand compaction pile improved ground as

$$\Delta c_t = K_1 \ (\Delta\sigma\mu_c) \cdot U_t \tag{23}$$

where:

Δc_t = increase in shear strength at time t of the clay due to consolidation

$\Delta\sigma$ = average increase in vertical stress in the unit cell on the shear surface due to the applied loading

μ_c = stress concentration factor in the clay, Equation (17a)

U_t = degree of consolidation of the clay at time t

46. The applied stress $\Delta\sigma$ includes the embankment loading and can be reduced, if required, to consider the spreading of stress in the sand compaction pile improved ground. A discussion of spreading of stresses in sand compaction pile improved ground (which is similar to stone column improved ground) is briefly discussed in the next section and in more detail byBarksdale and Bachus (1983a). Equation (23) gives a convenient method for estimating the increase in shear strength in normally consolidated cohesive layers at any time provided K_1 has been evaluated from field testing or estimated (Leonards, 1962). More sophisticated methods are available for considering increase in shear strength (Ladd and Foott, 1977).

47. If a soft cohesive soil is reinforced with sand compaction piles a reduction in settlement occurs (Ichimoto, 1980; Aboshi et al., 1979) with the magnitude depending upon the amount of stress concentration. To estimate the settlement reduction, assume that the vertical stress carried by the sand compaction pile and cohesive soil does not change with depth. This assumption is discussed in paragraphs 48 and 49. From conventional one-dimensional consolidation theory the settlement of the cohesive layer can be expressed as

$$\Delta H = m_v (\Delta\sigma_c)\ H \tag{24}$$

where:

ΔH = settlement of the compressible cohesive layer

m_v = modulus of volume compressibility determined from laboratory consolidation tests

$\Delta\sigma_c$ = average stress change in the cohesive soil due to construction, i.e., $\Delta\sigma_c = \mu_c \Delta\sigma$

H = thickness of the compressible layer

48. Calculation of settlement using Equation (24) or alternate expressions for settlement is straightforward for a site reinforced with sand compaction piles and subjected to a uniform loading over a very large area. Due to symmetry of loading and geometry due to the large extent of the area, the external load $\Delta\sigma$ applied over the area tributary to a given sand compaction pile remains for practical purposes in that tributary area

all the way down to the supporting strata. For this condition the average increase in stress in the cohesive soil $\Delta\sigma_c$ remains approximately constant with depth, and can be calculated using Equation (17a). An estimated value of the stress concentration factor based on past experience and field measurements is used in Equation (17a). The calculated increase in stress in the cohesive soil $\Delta\sigma_c$ is then used in Equation (24) to estimate the settlement of the cohesive layer.

49. For sand compaction piles covering an area of limited extent, a concentration of vertical stress still occurs in the compaction pile. With depth, however, the stress is gradually distributed outward to the unreinforced soft cohesive soil. The stress can be approximated using a Boussinesq stress distribution for a homogeneous soil (Aboshi et al., 1979). For small groups of sand compaction piles use of a constant vertical stress with depth would be overly conservative while use of Boussinesq theory would tend to somewhat underpredict the settlement. For stress distributions that vary with depth, settlement in a cohesive soil can still be calculated by dividing it into sublayers and using Equation (24) together with a suitable distribution of stress. Stress concentration effects are considered using Equation (17).

50. In calculating settlement the modulus of volume compressibility is usually taken as that of the cohesive soil before construction of the sand compaction piles.* Aboshi et al. (1979) report good settlement predictions using Equation (24). In Japan settlement estimates are routinely made using the above method in the soft clays. Settlements of 7 to 10 ft frequently are calculated when land is reclaimed from the sea. Aboshi et al. have found from field measurements that the settlement of sand compaction piles and the surrounding soft clay is approximately equal.

Conclusions

51. A considerable number of sand compaction piles, designed using the theory presented in this section, have been constructed and have performed satisfactorily. This theory is felt to be generally representative of the practice presently followed in Japan. As is common

* "Sand Pile Construction Using Vibrating Pile-Driving Equipment," Kensetsu Kikai Chosa Co., Ltd., anonymous (undated), Osaka, Japan.

elsewhere, some Japanese engineers apparently rely heavily on past experience and rules-of-thumb for the design of sand compaction piles. Specific site and loading conditions, past experience and usual design practices should of course always be carefully considered in selecting a final design.

PART IV: GENERAL DESIGN CONSIDERATIONS

Applications

52. The Japanese use extensively sand piles (i.e., sand compaction piles, mammoth compaction piles, and sand drains) for many different applications usually involving land that has been reclaimed from the sea. A summary of the detailed usage of sand compaction piles in Japan is given in Table 3. Fills, embankments, and tanks are routinely placed on very soft cohesive soils having shear strengths as low as 100 psf; construction is common on soils with shear strengths from 200 to 300 psf.

53. In reclaiming large areas of land from the sea, usually a dike is first constructed around the area to be reclaimed. To prevent a stability failure, the soft sediment is either dredged out and replaced with hydraulic fill, or else the dike is placed directly on the soft sediment after reinforcing it with closely spaced, mammoth compaction piles as shown in Figure 16. Mammoth compaction piles have the following advantages over conventional dredging and hydraulic replacement:

1. Since the sediments are not actually removed using the mammoth compaction piles method, disposal of waste sediment is not necessary. Elimination of waste disposal is highly desirable in Japan from the environmental viewpoint because of the high population density.

2. A denser sand can be obtained using the mammoth compaction piles method than for dumped hydraulic fill.

3. The vertical sand piles constructed by the mammoth compaction piles method require a minimum amount of clearance outside the proposed construction. In contrast, dredging necessitates the use of very flat side slopes in the soft ocean sediments requiring additional space outside the construction area.

4. The mammoth compaction piles method can be used adjacent to existing structures.

54. Mammoth compaction piles are frequently used in reclaiming land from the sea in Japan. Conventional dredging and replacement techniques, however, are still employed where environmental or other restrictions do not prevent their use. Other applications of this method include embankments, quay walls, piers, and breakwaters (Figure 17).

Table 3

Detailed Usage of Sand Compaction Piles in Japan

(Composer System, Fudo Construction Co., Ltd.,

Anonymous (undated), Tokyo, Japan)

Applications \ Clients / Types of Soils	Ministry of Construction	Ministry of Agriculture & Forestry	Japanese National Railways	Japan Highway Public Corp.	Japan Housing Corp.	Other Public Bureaus	Pref. Governments	Subways	Steel Makers	Petroleum Companies	Electric Power Companies	Shipbuilders	Miscellaneous	Ultra Soft Soils	Soft Clayey Soils	Loose Sandy Soils	Sandy and Clayey, Alternate	Deep Bearing Stratum	Special Soils	Market Volume
Prevention of Sliding Failure	●	○	○	●		○	○	○					○	●	○		□			●
Approach of Embankment	○		□	●			○							○	○		○			○
Vibration Resistance			○											●	○		○			○
Extra Heavy Rail Load									○					○	○	○				○
Sliding Resistance for Heavy Yards									○					●	●				○	●
Foundation of Tank										●	●		○		●	●	●		○	●
Foundation of Structure	□		○			○	○				□	○	○		□	○	○	○	□	○
Compaction of Cellular-fill						□	□		□		□				○	○				○
Sliding Resistance for Quay Wall						○	○												○	○
Horizontal Resistance of Ground																				□
Prevention of Settlement			○			○								○	○					□
Promotion of Settlement			○			○								○	○					□
Improvement of Road Base				□		□								□						□
Foundation of Building					○								○	□		○		○	□	○
Excavation, Prevention of Heaving							□		□						○					

●: very good ○: good □: special

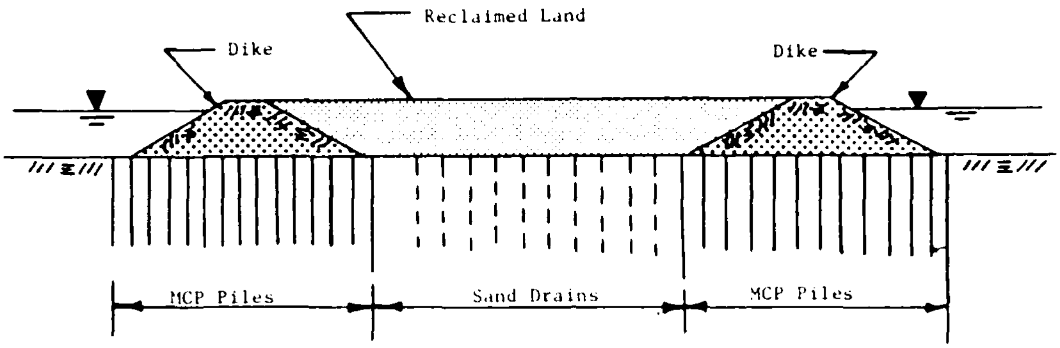

Figure 16. Land reclamation using mammoth compaction piles (MCP)

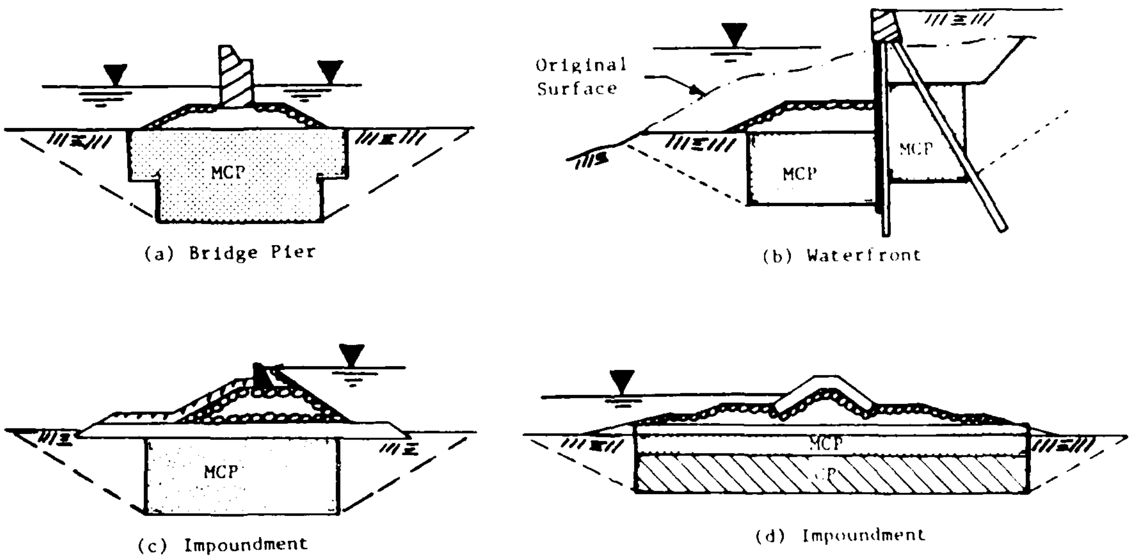

Figure 17. Typical application of the mammoth pile method ("Mammoth Compozer System," Fudo Construction Co., anonymous (undated), Tokyo, Japan)

General Criteria and Practices

55. Japanese design practices and design criteria for sand compaction piles for selected organizations are summarized in Table 4. Both square and triangular grid patterns are commonly used in Japan for all types of site improvement work. Sand compaction piles constructed on land have area-replacement ratios varying from 40 to 50 percent while sand piles constructed in the sea have area-replacements ratios varying from 20 to 90 percent depending on the application and soil strength. A higher replacement is usually employed for reclamation of land from the sea using the mammoth compaction piles method than for projects on land; for this type reclamation the piles often touch each other with typical area-replacement ratios of 70 to 80 percent. Finished sand compaction pile diameters generally vary between 24 and 32 in., and mammoth compaction pile diameters between 32 and 70 in. A pile spacing (pitch) is used between 4 and 10 ft. Typically, however, the spacing is between 5 and 7 ft. If a small sand compaction pile spacing is used, problems may be encountered in previously constructed adjacent piles, and special precautions must be taken.

56. Sand compaction piles become hard to install in soils having standard penetration resistances greater than about 15 to 20. To penetrate very dense sand layers and hard clay seams water jets having pressures from 70 to 1,200 psi are sometimes placed on the sides of the casing (Hayashi, 1981). Japanese engineers apparently prefer to carry sand compaction piles down to a bearing strata. This practice is in agreement with stone column design practice in the United States and Europe. However, when the bearing stratum is quite deep, a floating sand pile design is apparently used in Japan in some instances for stability applications (Mizutani, 1981).

Stability Considerations

Stability performance

57. The fact is well established in Japan that sand compaction piles, mammoth compaction piles, and strong sand piles have a record of good performance with respect to stability. All of the Japanese engineers interviewed pointed out that only one to two stability failures of these type piles have been reported in Japan. The absence of stability failures is

Table 4

Summary of Sand Compaction Pile Design and Construction Practices in Japan

Organization or Project	Type Organization/ Project	Stability Design				Construction				Soil Conditions	Air Press. (kg/cm²)	Sand	Production Rate	Cost	Comments
		φ (o)	n	a_s	SF	Casing Diam. (mm)	Pile Diam. (mm)	Spacing (m)	Length (m)						
Kensetsu Kikai Chosa Co. Ltd.	Manufacturer of Vibratory Pile Driving Equip.	20-45 45° for coarse clean sand	4-5	-	1.1	-	-	1.7-2.0	-	-	3-5	-	-	-	Consolidation; 80% during construction (following Civil Engr. & Geot. Assoc. guidelines); *End support desirable - for stability not necessary; sand pile <15% silt*
Fudo Const. Co., Ltd.	Large Construction Company	30 (it may be 35°)	3	-	1.2-1.3	500	800	1.2-3.0 Typical: 2 Sea-may touch, 1.6-3.0 LNG Tanks	-	-	3-5	-	-	-	Always end support Sand for Piles: <10% silt; 0.01 to 1 mm sand
Nippon Kokan Co., Ltd.	Large Steel Company	30° dumped 40° vib. (N=20-25) May be 45°	3(4) 10(4)	-	1.25	400	700	2-2.5	-	-	-	-	-	-	Consolidation: 80% over 8 months to 1 yr. (for 2-3 m settlement) Pressure: Iron Ore Yard: 4000-6000 psf with 2 to 2.5 m pitch Go if possible to end support
Waste Disposal Station - Tokyo Bay - 6000 M P Piles	Contractor Nippon Kaikou (Soil Improvement Co.)	-	-	0.79	-	800	1200	1.20 □	24	24m - v.soft to firm silt (N<5); 10 m firm silt (N=5); 16 m water	7(1)	Clean fine to med. sand	21 piles day for 3 vibrators (13 hr. shift)	-	Vertical Vibrator: 120 kw (KM2 - 24,000A) 450 KVA Generator Bearing top of firm silt
Apartment Bldgs. Osaka Bay 1384 SSP Piles	Contractor Nippon Kaikou	-	-	0.27-0.49	-	800	Variable 1350(bot) 1000(top)	1.70 □	28-32	5-6m - v.loose sand; 23-26 m v.soft clay N=0; Gravel supporting layer; 2-3 m water	3-4	Clean coarse sand with shells	45 piles/ day for 3 vibrators	$770/ pile	Vibrator: 300 kw (KM4 - 48,000A) 450 KVA Generator Horiz. Vib.: SSP 7.5 kw Casing pulled down using cable and winch; water added to sand
LNG Storage Tank Toyoko Bay 58.2 dia. 1069 SC Piles	KAJIMA Corp. General Contractor	-	-	0.11	-	400	600	1.70 △	6	3 m sand (N=4-9) 1 m silt (N=4); 2.5m sand (N=11-34)	7(1)	Clean fine-med. sand	60 piles/ day avg.	-	Vibrator: KM2 - 12,000A (90 kw) 3840 psf area loading; 3 m preload for 3 months
LNG Storage Tank	Shimizu Co. General Contractor	-	-	0.11	-	400	650	1.70 □	15	40 m fine sand with silt	7(1)	Med. sand <5% silt	20 piles/ day avg.	15 m - $196(2)	Vibrator: KM2 - 12,000A (90 kw) Lift casing 3 m; repenetrate 1.5 m 40t crane; 194HP Compressor
Oil Tank Factory Kaohsiung City Taiwan	Chou San Development Enterprise Co., Ltd.	-	-	0.08	-	400	600	2.0 △	8-12	8-10 m well-graded gravel with sand (N=8-10) w. 3 clay layers; sand	Yes	Well-graded sandy gravel	20-40 piles day/ machine (25 avg.) 8:00 a.m. to 10:00 p.m. (2 shifts)	-	4 Rigs: 1 - 120 kw, 3-90 kw Pile Installation: 6 - 7 min./pile Compaction: Extraction - 3.7 m, Repenetration 2.7 m Sand Piles: For liquefaction - concrete piles installed

Notes. 1. Air pressure was apparently given at compressor and not top of sand pile.

2. Cost of just the sand; cost of sand compaction piles is one-half the cost of a precast concrete pile, driving costs are about the same.

3. Includes $7.35/m for cost of sand.

rather remarkable considering the widespread use in Japan of sand compaction piles for this purpose.

58. Aboshi et al. (1979) have described a well-known trial embankment constructed on 36 ft of organic silt having a shear strength of 200 to 300 psf. The organic silt was underlain by 95 ft of peat with a shear strength varying from 150 psf to 250 psf. A test section without sand compaction piles failed at an embankment height of 21 ft. Another section of the embankment was constructed using a square grid of sand compaction piles at a 6.6-ft spacing resulting in an area-replacement ratio of only 0.1. Nevertheless, failure did not occur under an embankment height of 48 ft. The exceptional performance of this embankment suggests why the primary use of all types of sand compaction piles in Japan is to prevent stability failures.

59. The method of stability analysis previously presented has been found in Japan to give conservative results for embankments constructed on sites stabilized with sand compaction piles (Aboshi et al., 1979). In 15 test embankments, embankment loads between 1,600 and 7,100 psf were supported by sand compaction piles having area-replacement ratios between 0.16 and 0.20. Calculated safety factors of these embankments with respect to a stability failure varied from 0.99 to 1.59 with seven of the embankments having safety factors less than 1.10.* Nevertheless, none of the embankments failed even though over one-half of them were constructed over soils having shear strengths between 120 and 300 psf.

Angle of internal friction

60. The angle of internal friction ϕ_s of sand compaction piles used for stability analyses in Japan appears to vary from 30 to 40° depending upon the organization (Table 4). Fudo Construction Company, Ltd. routinely use $\phi_s = 30°$ for design although it is admitted that ϕ could be as great as 35°. Nippon Kokon Co., Ltd., a very large steel company, uses 30° for dumped sand and 40° for vibrated sand having an N value of 20 to 25; for this condition they feel the angle of internal friction may be as large as 45°. Kensetsu Kikai Chosa, a well-known Japanese manufacturer of vibratory hammers, recommends angles of internal friction between 20° and 45°

* It is not clear if consolidation of the foundation soils during construction was considered in the stability analyses. Undoubtedly long-term consolidation helps to explain the success of these embankments.

depending upon the material; for a coarse clean sand a value of 45° is recommended. The validity of using angles of internal friction of 40° or more for design is not recommended by the writer. Such high friction angles would require a very small void ratio and coarse, angular particles.

61. For sand compaction pile design in the United States, a reasonably conservative value of the angle of internal friction should be employed in stability calculations at least until sufficient experience has been gained using this technique. For sand compaction piles constructed using sands with less than 8 percent silt content, the recommendation is made that an angle of internal friction ϕ_s between 30° and 38° be used depending upon the effective grain size, gradation, and densification attained. The angle of internal friction used in design should of course be no greater than the value of ϕ_s estimated from the design relative density and/or standard penetration resistance obtained from field measurements and corrected for overburden pressure.

62. The writer recommends a safety factor with respect to a stability failure of at least 1.3 and preferably 1.5. It is recognized that the sponsor (CE) has its own criteria on safety factors for slope stability analyses defined in EM 1110-2-1902 US Corps of Engineers, 1970. The actual safety factor selected should depend upon a number of factors including (a) the specific problem including site conditions and type construction, (b) whether an increase in shear strength due to consolidation has been considered, and (c) existing criteria of the design agency. In Japan a safety factor of 1.2 to 1.3 is commonly used for stability applications.

Local bearing failure

63. Sand compaction piles, when used for stability applications, carry reasonably large shear forces (but not as large as stone columns). These shear forces could conceivably result in the sand compaction pile punching into the surrounding very soft soil although this type failure has not been reported in Japan. A local bearing (punching) type failure was observed in the US at Jourdan Road Terminal during a direct shear test on a stone column (Barksdale and Bachus, 1983a). Stone columns in general would be more susceptible to a local bearing type failure than sand compaction piles since they are usually designed to carry higher axial and hence shear loads (i.e., stone columns are usually designed using a greater angle of internal friction). Nevertheless, the possibility of a local

bearing failure does exist for sand compaction piles constructed in extremely soft sediments having undrained shear strength less than about 250 psf. A discussion of local bearing failure, together with a presentation of design theory and design charts has been given elsewhere (Barksdale and Bachus, 1983b).

Stress Concentration

64. The value of the stress concentration factor n used has an important effect on both stability and settlement calculations. For a test embankment at Fukuyama the measured stress concentration factor, which was almost constant with depth, increased from 1 to about 4 as the 13-ft-high embankment was placed (Aboshi et al., 1979). After the embankment was completed the stress concentration factor continued to increase up to about 5 as consolidation occurred. Further, the average stress concentration factor n measured at twenty sand compaction pile sites in Japan underlain by soft clay is 4.9, with observed values for the twenty sites varying from 2.5 to 8.5.* An n value of 4 was found to give the best agreement with the measured time rate of settlement curve for one test embankment.

65. In estimating the magnitude of stress concentration the reader should remember that theory shows n to increase for (a) increasing values of the modulus ratio of the sand column to the in situ soil and (b) for decreasing area replacement ratios (Ogawa and Ishido, 1965). For design Fudo Construction Co., Ltd. uses a stress concentration factor n of 3.0. Nippon Kokan Co., Ltd. uses n = 3.0 for dumped sand and n = 10.0 for vibrated sands having an N value between 20 and 25. Kensetsu Kikai Chosa recommends an n value between 4 and 5. For the present time a stress concentration factor n of 2 (certainly no more than 3) is recommended for stability applications of sand compactiion piles constructed in the United States. For settlement calculations a value of n between 3 and 4 is suggested.

*The stress concentration factors cited neglect the extreme values measured.

Sand Pile Gradation

66. Sand is usually used in Japan for site improvement work since it is the most readily available material. Gravel and even crushed stone have, however, been used on a limited basis in Japan and Taiwan. For site improvement using sand compaction piles, the selection of a proper sand gradation appears to be even more important than for the much larger size aggregate used in stone columns. The important beneficial effect of increasing sand size on standard penetration resistance after densification is illustrated in Figure 18.

67. Saito (1977) has presented extensive field data showing the important detrimental effect that fines have for sands improved using the vibro-rod technique (Figure 19). The vibro-rod method is used to densify sands following a procedure somewhat similar to the Terra Probe (Brown and Glenn, 1976). The vibro-rod consists of a closed pipe having a number of outward protrusions that is driven by a vertical vibrator located at the top of the pipe. Saito found as the fines content of the sand increases up to about 15 percent, the standard penetration resistance after construction decreases exponentially. Above about 15 percent fines, vibration was found to have little beneficial effect on the standard penetration resistance of the soil. These findings clearly indicate that specifications for sand compaction piles should require a clean sand with very little, if any, fines.

68. Sand gradation specifications from five Japanese government harbor agencies are given in Figure 20. Figure 21 shows actual sand gradations used to construct mammoth compaction piles over water at four land reclamation sites. All of the sand specifications called for well-graded, fine to medium sands which do not have minus 0.05 mm sizes. Somewhat lower quality sands are apparently also sometimes used which have less than 10 percent fines, and grain sizes varying from 0.0004 to 0.04 in. as indicated in Figure 21.

Quality Control

69. Specifications in Japan for sand compaction pile construction usually require a minimum standard penetration resistance of either the

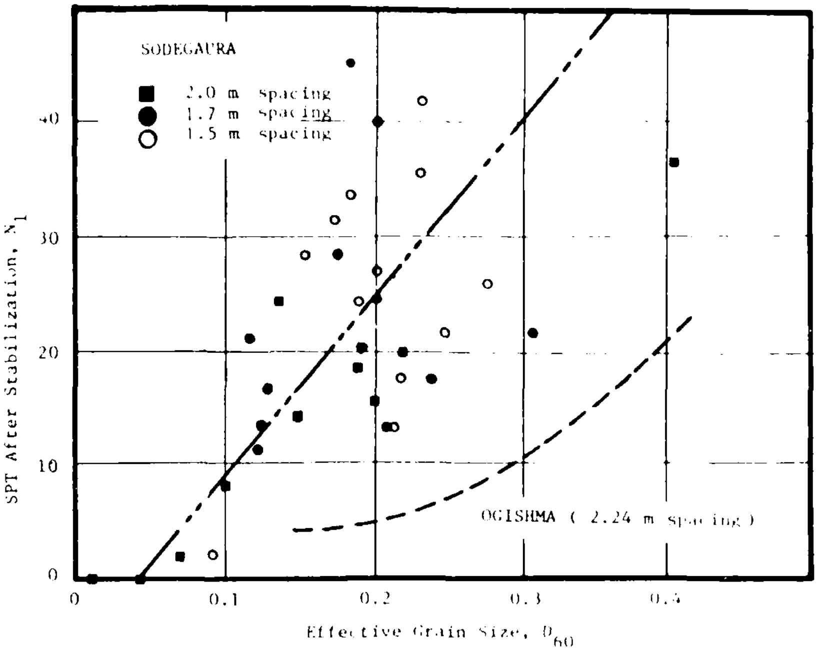

Figure 18. Influence of grain size on standard penetration resistance after densification--strong sand pile method ("SVS Method Technical Information," Kensetsu Kikai Chosa Co., Ltd., Osaka, Japan)

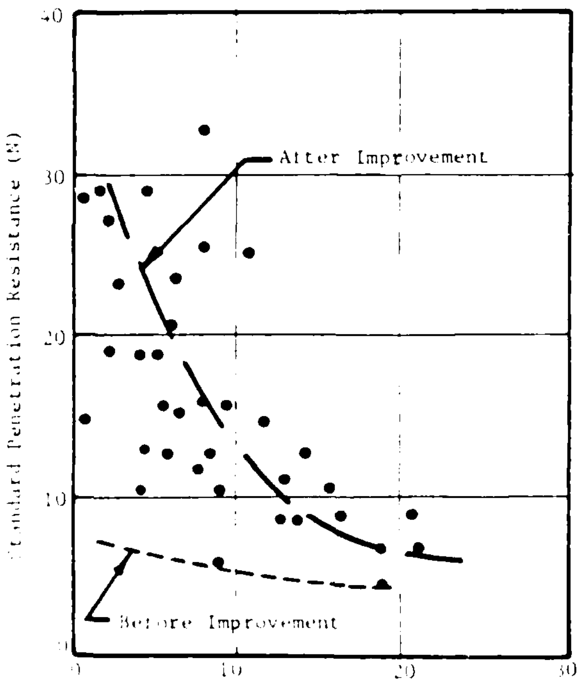

Figure 19. Influence of fines content on standard penetration resistance--vibro-rod method (after Saito, 1977)

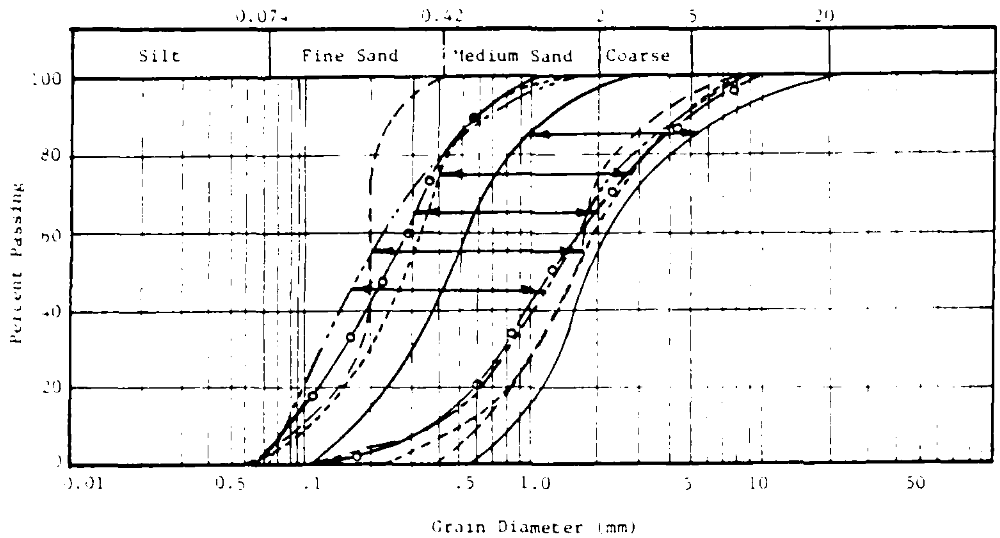

Figure 20. Sand gradation specifications used by five harbor agencies

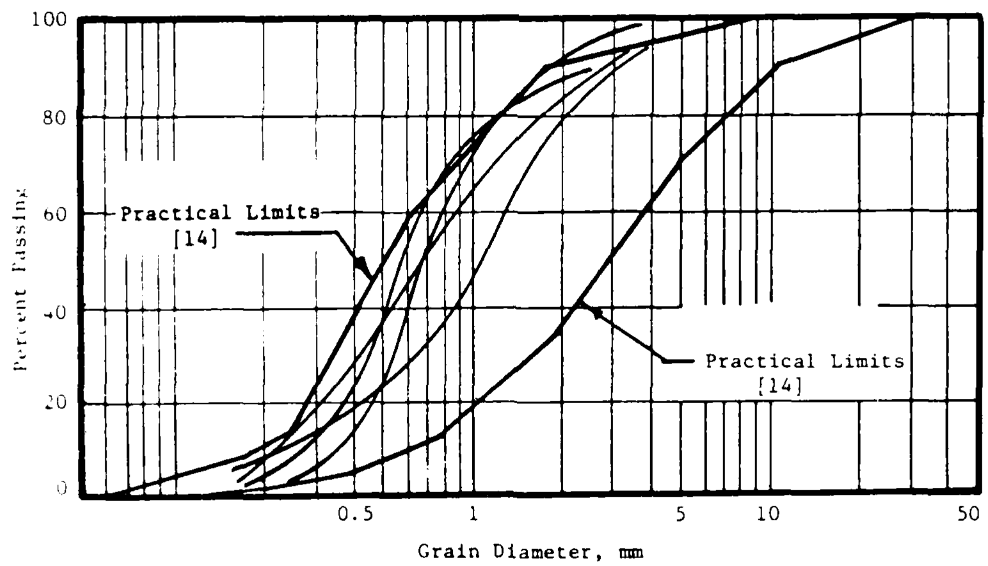

Figure 21. Sand gradations used for four mammoth compaction pile jobs ("Mammoth Compozer Systems," Fudo Construction Co., Ltd., Tokyo, Japan)

sand pile or the in situ soil at sites underlain by cohesionless materials. For sand compaction piles constructed in cohesive soils the penetration resistance is usually measured down the center of the pile. Specifications usually require a standard penetration resistance N value between 10 and 20.* At sites underlain by sands, the penetration resistance between compaction piles is usually required to be greater than 10 to 15. As shown in Figure 22, an average N value of 18.9 was observed for a large number of mammoth compaction pile jobs with the standard deviation being 5.4.

70. An indication of the current practices in Japan of specifying sand pile construction work can perhaps be best obtained by briefly describing the practices followed at the four sites visited. For the construction of two LNG tanks at separate sites underlain by sand, specifications required N to be greater than 15 at the center of the sand compaction pile grid; one site used a square grid and the other a triangular grid. At a waste disposal station site being reclaimed from the sea, three borings were put down in every third mammoth compaction pile; originally one boring was to be placed in each pile. The specifications on this job required a standard penetration resistance greater than 15 in the center of each pile. At a land reclamation site for apartment houses, 3 or 4 test borings were to be made for a total of 1,584 piles. Specifications required N to be greater than 10. Typical values of N measured at this site were 4 to 5 at a depth of 10 ft, 20 ft at 39 ft, and 30 at 75 ft. As expected, the number of piles tested varies considerably depending upon the specific application and designer. As a general rule-of-thumb Fudo Construction Co., Ltd. usually tests one pile for every 3,280 lin ft of pile constructed.

71. In developing specifications for sand pile construction work, the effect of overburden pressure should be considered in developing required values of the penetration resistance in the field. At the present time it is apparently more usual in Japan to not consider the effect of overburden pressure for quality control, although many Japanes engineers recognize this need and undoubtedly some correct for overburden pressure.

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* Apparently the specified N value is usually uncorrected for overburden pressure.

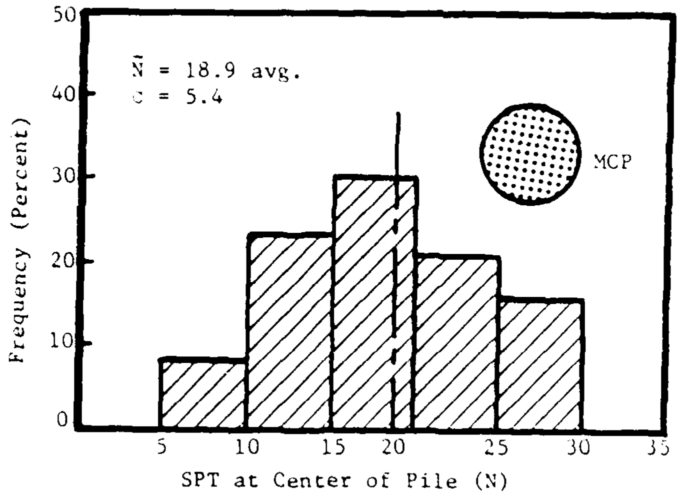

Figure 22. Observed variation in SPT value for a large number of mammoth compaction pile ("Mammoth Compozer System," Fudo Construction Co., Ltd., Tokyo, Japan)

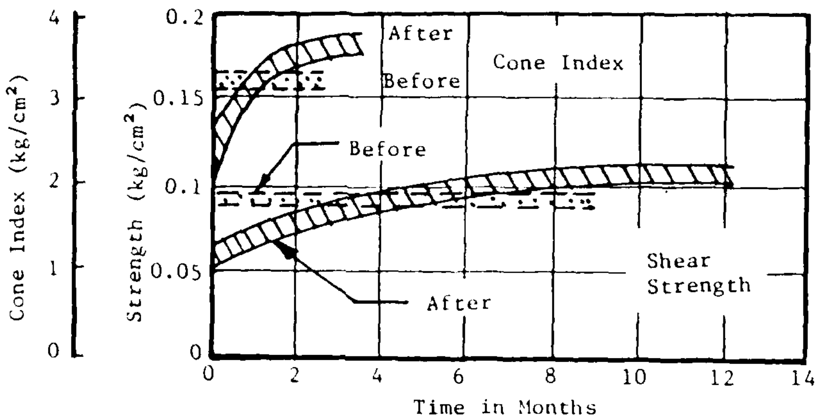

Figure 23. Effect of sand pile construction on shear strength as a function of time (after Ichimoto, 1980)

Influence of Lateral Pressure on SPT Value

72. Construction of sand compaction piles typically results in standard penetration resistances increasing by a factor of 2 to 5 as shown in Figure 13 and 14. Based on extensive field measurements made at the Ogishima Island Steel Plant, Saito (1977) has concluded that the large measured increase in standard penetration resistance is not accompanied by a corresponding large increase in relative density. As a result, the relative density obtained when the measured N value is corrected for overburden pressure was found to considerably over-estimate the actual relative density of the improved soil at the Ogishima site. The misleadingly large standard penetration resistances measured after site improvement is attributed to the significant increase of lateral pressure during densification of the sand. Pressuremeter measurements indicated an increase in K_o from about 1 to as much as 6. The important effect of lateral stress on standard penetration resistance has been previously reported (Rodenhauser, 1974; Zolkof and Wiseman, 1965; Schmertmann, 1975). More research is certainly needed to develop techniques for handling the effect of lateral pressure increase on relative density.

Strength Loss Due to Sand Pile Installation

73. Construction of sand compaction piles in cohesive soils causes a significant loss of strength due to remolding of the soil surrounding the pile (Figure 23). As pore pressures dissipate the shear strength of the clay gradually increases, with the time required to regain its initial strength depending to a great extent upon the physical-chemical characteristics of the soil. Field measurements in Japan indicate that typically from 2 to 20 weeks are required to regain initial strength with 2 to 3 weeks perhaps being more usual (Ichimoto, 1980; Aboshi et al., 1979). For mammoth compaction piles the effect of remolding is even more severe than for sand compaction piles, but strength gain has still been observed (Ichimoto, 1980).

74. Ichimoto (1980) recommends waiting in cohesive soils at least a month before loading sand compaction piles to allow at least a partial

recovery of initial strength. Further, when constructing sand compaction piles in granular soils, the existance of a clay seam or even a relatively impervious fine sand seam will prevent the N-value from increasing until some time after construction. Therefore, if low permeability layers are present the standard penetration resistance should not be measured until as long as possible after construction (Ichimoto, 1980).

75. In constructing sand compaction piles in very soft clays, loss of strength due to remolding can be an important problem. To overcome this problem, sand having a high angle of internal friction should be used or else the construction should be organized in stages (Ichimoto, 1980). For stage construction, first place the piles at twice the final spacing and later, after strength gain has occurred, install a second series of piles midway between the piles previously constructed.

76. Sand compaction piles are vibrated into the ground. As a result important shear stresses are transferred to the surrounding soil. Also "smear" of the native soil occurs adjacent to the sides of a sand compaction pile. Field studies show that when a pipe is pushed into the ground without jetting, a significant reduction in lateral permeability occurs; the effect of smear is much less; however, when the pipe is jetted into the ground (Barksdale and Bachus, 1983(a); Casagrande and Poulos, 1968). Since stone columns are usually jetted into the ground, smear effects and the remolding of the surrounding soil is much less than in sand compaction pile construction.

PART V: SUMMARY AND CONCLUSIONS

77. The primary use of sand compaction piles in Japan is to prevent stability failures. When settlement is of concern, preloading is often used after the sand compaction piles have been constructed. Sand compaction piles are used extensively in Japan to support dikes, embankments for roads and railways, stockpiles of heavy materials and tanks. Embankments are routinely successfully constructed on cohesive soils with shear strengths varying from 200 to 300 psf that have been reinforced with sand compaction piles.

78. Sand compaction piles are constructed by driving a steel casing (pipe) filled with sand to the desired elevation using a heavy vertical vibrator placed at the top of the pile. The casing is then gradually extracted using a stroking motion to densify the sand. Using this construction sequence, soft cohesive soils surrounding the pile are not left partially unsupported as is usually the case during the construction of stone columns. Also sand compaction piles are constructed by dumping sand down the casing and not down an uncased hole as occurs during stone column construction. Since jetting is not normally used, construction of sand compaction piles does not result in a large quantity of excess muddy water which is often environmentally objectionable in stone column construction. Erosion of fines into the sand compaction pile should not be a problem for normally used gradations of sand. Finally, construction of sand compaction piles, which is fast and efficient, utilizes low-cost, often locally available sand.

79. The important disadvantages of using sand compaction piles to prevent stability failures of embankments appears to be (a) sand has a lower angle of internal friction than the large size stone used in stone columns and hence requires a larger area replacement ratio, (b) smear and remolding effects are greater for sand compaction piles than stone columns and, (3) contractors in the United States do not have either the experience or mechanized equipment for rapidly constructing sand compaction piles. Finally, sand piles would not act as vertical drains during an earthquake. For many applications these disadvantages are not serious. Several different vertical vibrators are presently available in the United States that could be used to construct sand compaction piles. The automatic skip

loading system used in Japan and Taiwan could be imported from Japan, or else a similar system (or even a simple version) could be designed and constructed in the United States.

80. Typically approximately 80 percent of the consolidation settlement occurs in a cohesive soil reinforced with sand compaction piles within about 8 months from the beginning of construction. Therefore, an important part of the settlement due to an embankment should be complete by or shortly after the end of construction; consolidation of stone column improved ground should progress somewhat faster. Because of the lower cost of sand and faster production rates, sand compaction piles constructed in the United States using efficient equipment similar to that employed in Japan should cost about 1/2 to 2/3 the cost of stone columns. For many applications involving, for example, stability of embankments, sand compaction piles offer a quite viable alternative to stone columns. Therefore on appropriate projects alternate bids and/or demonstration type projects should be used to encourage the development of sand compaction pile technology in the United States.

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